GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

SAN FRANCISCO, CALIFORNIA

REPORT ON FEASIBILITY OF INSTALLATION OF A LOWER DECK FOR AUTOMOBILE TRAFFIC ON GOLDEN GATE BRIDGE

AMMANN & WHITNEY

Consulting Engineers

August 1964

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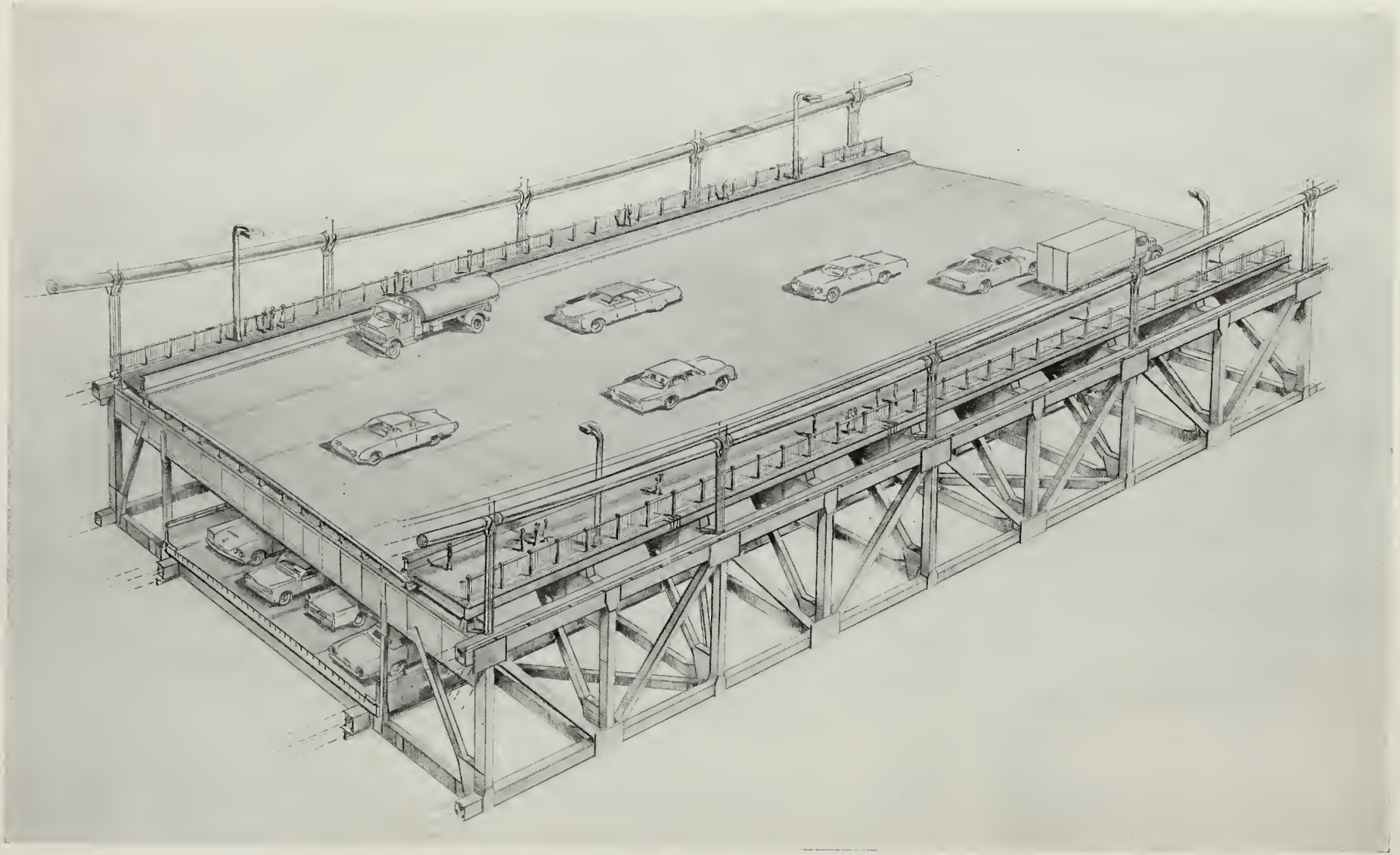

PERSPECTIVE SHOWING INSTALLATION OF A LOWER DECK FOR AUTOMOBILE TRAFFIC ON GOLDEN GATE BRIDGE

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August 6, 1964

To the Honorable Board of Directors
Golden Gate Bridge & Highway District
Box 9000, Presidio Station
San Francisco, California

Gentlemen:

In accordance with our agreement of June 19, 1964 we submit herewith our report on a study of the feasibility of adding a second deck to the Golden Gate Bridge to carry four additional lanes for passenger automobiles and other light vehicles.

This study and report was to be limited to the suspension bridge proper from anchorage to anchorage and passage through or around the existing anchorages.

As a result of this study we have come to the conclusion that the construction and operation of such a lower deck is entirely feasible, that aside from the addition of the lower deck only moderate changes in the existing structure are necessary, that such changes can be carried out without interference with the flow of traffic on the existing upper deck and that it would leave the structure in a conservatively safe condition.

We wish to express our appreciation of having been retained to make this study for which we had the very helpful assistance of your General Manager, Mr. James Adam, and his staff.

Respectfully submitted,

AMMANN & WHITNEY, Consulting Engineers

Boyd G. Anderson

By B. G. ANDERSON, C9982

O. H. Ammann

By O. H. AMMANN

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GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT

REPORT ON FEASIBILITY OF INSTALLATION OF A LOWER DECK FOR AUTOMOBILE TRAFFIC ON GOLDEN GATE BRIDGE

SUMMARY OF CONCLUSIONS

This study leads to the conclusion that the addition of a lower deck on the suspension bridge to carry four lanes of automobiles and other light vehicles is entirely feasible with very moderate changes in the existing structure.

The additions and changes can be accomplished without interference with the normal flow of traffic on the upper deck.

Because of considerable reserve strength in the major carrying parts of the existing structure, the cables, anchorages, towers and stiffening trusses, it is feasible to add the proposed lower deck for automobiles with relatively very small increases in the stresses. The resulting stresses will remain within conservatively safe limits, even with allowance for possible moderate future structural additions if that should ever become necessary or desirable.

The existing 60 ft. wide six-lane roadway, which accommodates mixed vehicular traffic, has become substandard and its widening is not possible. It is therefore proposed to operate it with five lanes after the lower deck is built. This widening of the lanes will decrease the traffic hazards and tend to speed the traffic flow. The traffic capacity of the two decks in number of vehicles which can cross the bridge in peak hours should be increased by up to 85 percent of that of the present structure.

It is estimated that the addition of the second deck will extend the time to reach the limit of traffic capacity of the bridge by 20 years or more.

A general study of the structure over the anchorages and of the approach viaducts indicates the feasibility of the addition of a lower deck, but further detailed studies will be necessary to determine the extent of such changes. Parts of the approach viaducts may require extensive modifications, depending upon plans for extended highway connections.

The report includes a comparison of a lower deck for a four-lane automobile roadway with that for two tracks of rapid transit and summarizes the reasons why the former is permissible whereas the Engineering Board of Review appointed by the District in their report of July 1962 advised against installation of rapid transit.

We wish to emphasize that the scheme for increasing the traffic capacity of the bridge on which this feasibility study is based involves the least amount of changes in the existing structure and can be carried out at minimum cost and with no interference with the flow of traffic.

Further studies might disclose other schemes by which the traffic capacity and the time to traffic saturation can be further increased. Such schemes, if found feasible and economically justified, would undoubtedly involve more extensive and costly changes in the present structure and might not avoid some interference with the flow of traffic during peak hours.

TRAFFIC CAPACITY OF EXISTING SIX-LANE UPPER DECK AND EFFECT OF ADDITION OF LOWER DECK

The existing roadway is 60 ft. wide between curbs and accommodates six lanes of mixed vehicular traffic. At times of one-way peak traffic four lanes are used for traffic in that direction and two lanes for the opposite direction.

According to the report of October 1963 by Arthur C. Jenkins, Consulting Engineer, the morning peak traffic on four lanes reached 5,425 vehicles per hour in 1963, or practically saturation which Mr. Jenkins estimates at 5,400 vehicles. The evening peak hour traffic on four lanes reached 5,283 vehicles per hour in 1963, only slightly below saturation.

The two lanes used by traffic in the opposite direction have carried at the time of the four-lane hourly peak traffic in 1963 up to 1,547 vehicles per hour or about 22% of the total traffic. This is well below the practical capacity of two lanes. The present limit of capacity of the bridge is therefore determined by that of four lanes of mixed traffic in one direction.

The existing 60 ft. roadway cannot be widened. Furthermore, it has become substandard for six lanes of mixed vehicular traffic. The present standard of the American Association of State Highway Officials calls for 13 ft. outer lanes and 12 ft. inner lanes, or 74 ft. for a six-lane roadway.

It is therefore proposed that, if a four-lane lower roadway for automobiles is added, the existing upper roadway be used for five lanes of mixed traffic, the two outer lanes being made say 12′-6″ wide and the three inner ones 11′-8″, which comes close to the present standard. This roadway could then be operated with three lanes in the peak direction and two in the opposite direction. This arrangement would unquestionably improve the safety of traffic.

The proposed lower deck would have a 44 ft. wide roadway for four 11 ft. wide lanes for automobiles and other light vehicles only. This limitation fits

well into the traffic pattern. Records on the Golden Gate Bridge and other large bridges in Metropolitan areas indicate that during peak hours more than 90% of the vehicles which pass over the bridge are automobiles.

The lower roadway could be operated either with two lanes in each direction or with all four lanes in the direction of the peak traffic. Thus, with three lanes on the upper deck, the total number of lanes available for the peak direction would be increased from four to seven lanes or by 75%. Traffic in the opposite direction would have two lanes for mixed traffic as at present.

The proposed widening of the lanes on the upper deck and the addition of four 11 ft. lanes for autos on the lower deck will tend to increase the capacity in number of vehicles passing the bridge. The total increase in this capacity of the bridge is estimated to be about 80%.

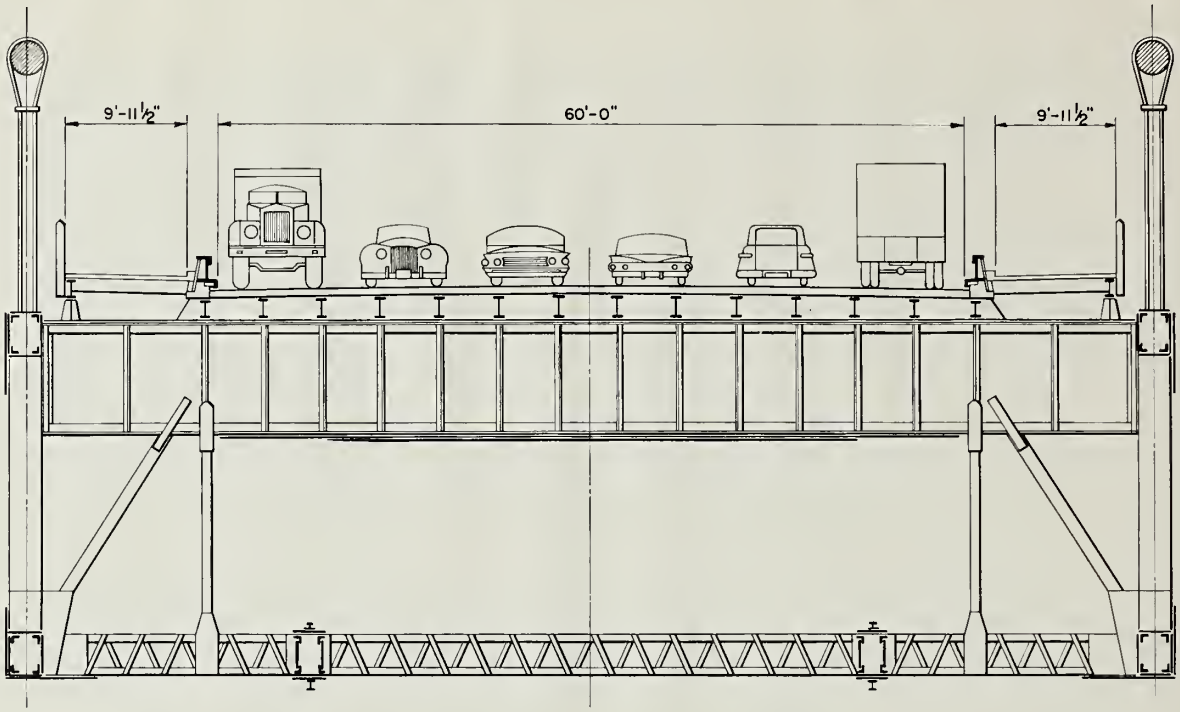
Future increases in actual traffic volume cannot be forecast closely. According to Mr. Jenkins' report the average daily traffic was 60,275 vehicles in 1963. Estimates indicate that by 1985 the number of vehicles which will cross the Golden Gate Bridge is in the neighborhood of 100,000.

DESIGN OF LOWER DECK AND OTHER CHANGES IN THE EXISTING STRUCTURE

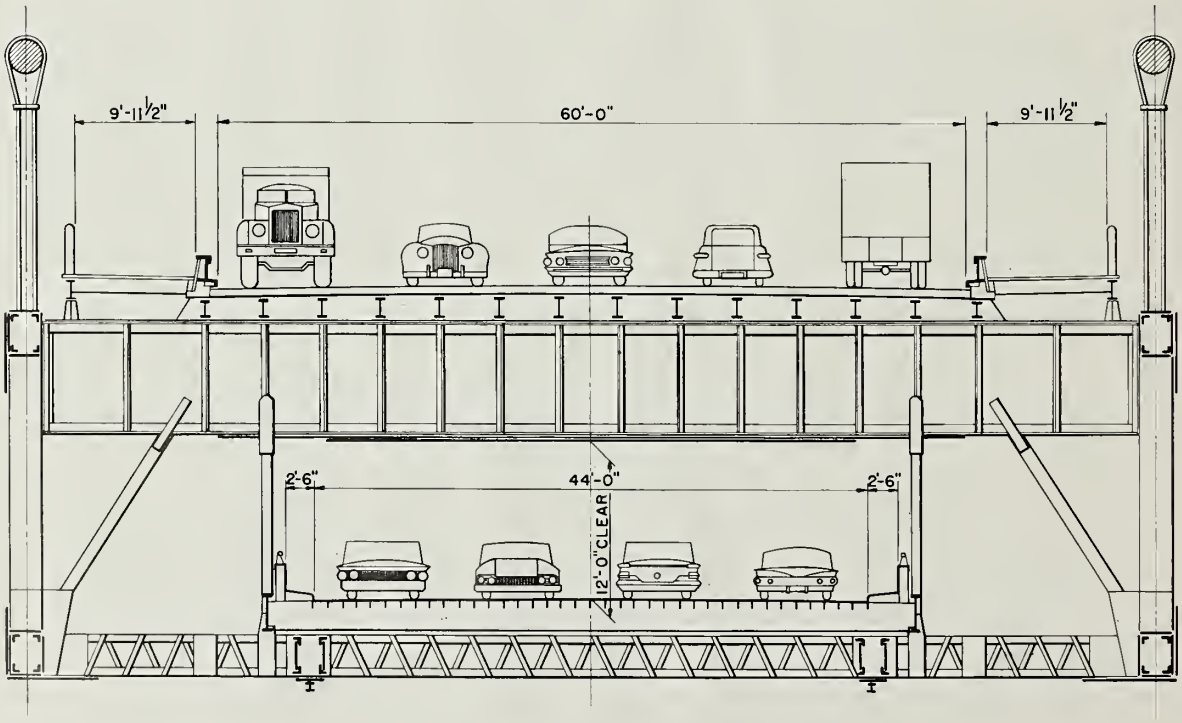
The proposed design of the lower deck, as illustrated by the attached typical cross section, is very simple. The carrying steel frame consists essentially of longitudinal stringers connected to and supported by vertical hangers between the upper and lower decks. Crossbeams about 6 ft. apart are framed into the stringers. Secondary plate stringers about one foot apart span the space between the crossbeams and, together with the latter, support a solid steel plate which forms the base for the paving. This entire steel work of the lower deck is of welded construction. It can be prefabricated in convenient sections to be shipped to the site for erection.

The existing hangers between the two decks will be replaced with new hangers shifted about 5 ft. toward the center line of the bridge in order to minimize the span of the deck crossbeams.

For the paving a one and one-half inch thick special asphaltic compound was assumed. Such a paving only one inch thick on a steel plate has been in use on two bridges of the Triborough Bridge and Tunnel Authority for nearly a year and has been found to stand up successfully under very heavy truck traffic. The assumed paving can therefore be expected to be eminently suitable for the light vehicles which will use the lower deck of the Golden Gate Bridge.



EXISTING DECK STRUCTURE
CROSS SECTION



PROPOSED DECK STRUCTURE
CROSS SECTION

GOLDEN GATE BRIDGE

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NEW YORK, N. Y.

DATE: AUGUST, 1964

However, further studies in connection with final plans may develop other types of surfacing as well as other designs for a light supporting steel structure.

An eight inch high steel curb, a service walk 2′-6″ wide, and a strong barricade are provided on each side of the 44 ft. wide roadway.

Structural Modifications of the Existing Suspension Structure

Aside from the addition of the lower deck structure and the shifting of the hangers between the two decks, changes in the main carrying parts of the existing structure are of relatively minor extent.

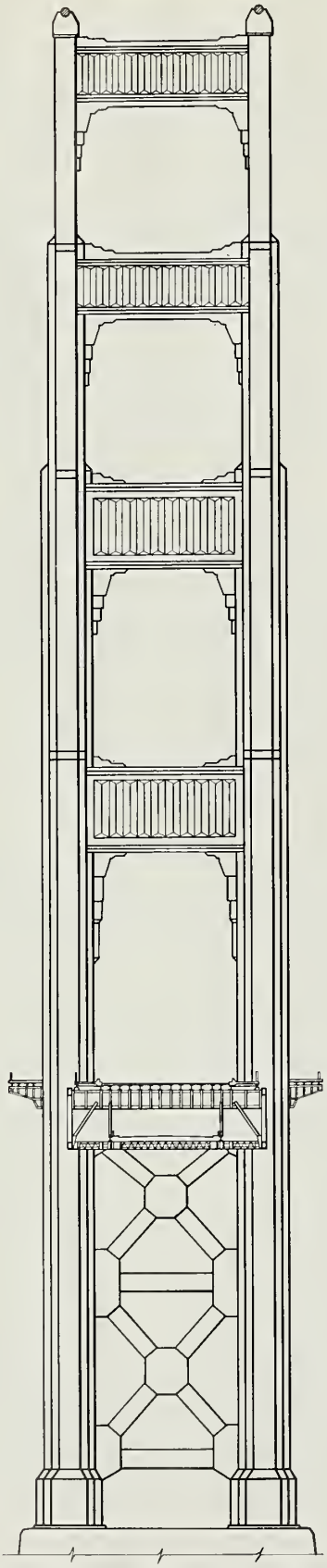
No changes are necessary on the cables, their anchorages and their supports at the pylons. Reconstruction work in the pylons and the Marin anchorage housing will be in areas unrelated to the main carrying members. The stiffening trusses, the upper lateral bracing system and the upper roadway, including the curbs and expansion joints, will be left intact. The lower deck will be in a position sufficiently above the existing lower lateral bracing to permit inspection and painting of all exposed surfaces. The transverse struts of the latter are affected only by the shifting of their connections with the new hangers between the two decks.

The contemplated erection schedule for the lower deck envisages its addition before any work is done on the upper deck. After the lower deck is in place the proposed modification of the upper deck sidewalks can be performed without interference with normal flow of traffic on the upper deck.

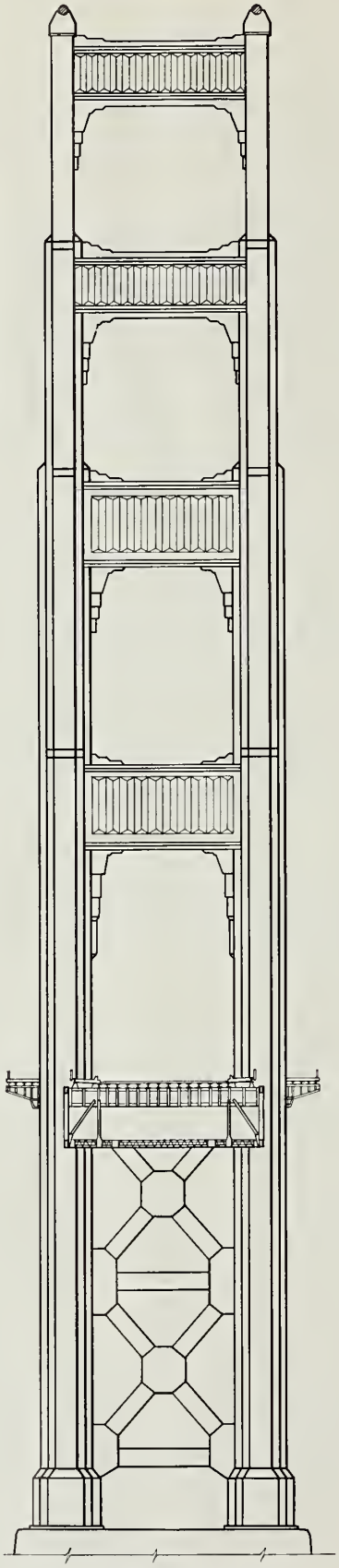
In order to reduce the dead load of the existing structure it is proposed to reconstruct the existing two ten foot sidewalks by using light steel construction with an asphaltic wearing surface or a light weight material such as aluminum. This lighter construction would still be of sufficient strength to accommodate light maintenance vehicles.

The existing interior traveling maintenance platform, as well as the rails supporting it, will be eliminated. Maintenance of the underside of the upper deck and other interior areas can be performed from the proposed lower deck. The existing exterior traveling maintenance platform and its supports will remain unchanged.

The floor trusses and floorbeams immediately adjacent to the pylons and the towers will require some structural modifications to provide the necessary clearance for the passage of vehicles on the lower deck. This work will be performed underneath the existing upper deck with traffic flow unimpeded.



ELEVATION OF TOWER
SHOWING LOWER DECK



ELEVATION OF
EXISTING TOWER

GOLDEN GATE BRIDGE

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NEW YORK, N. Y.

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No modification of the towers is necessary with the exception of the upper halves of the practically hidden diagonals immediately below the upper deck. These halves are to be replaced by similar members at a slightly flatter angle with the horizontal, as shown on the enclosed transverse elevation of the towers. This change when viewed in perspective will be hardly noticeable, if at all detectable.

The central part of the reinforced concrete walls of the pylons will require reconstruction to permit the installation of the lower deck. All work will be performed without in any way affecting the upper deck structure. As the proposed lower deck lies wholly within the confines of the stiffening trusses and above the existing lower lateral bracing the modifications in the pylon walls referred to above will not affect the architectural appearance of the pylons.

For similar reasons the architectural appearance of the Marin anchorage housing and the steel arch span on the San Francisco side will not be changed. All structural modifications will be made in the upper and central parts of these structures with no effect on the existing upper deck.

The increased dead load on the bridge will have the tendency to lower the stress in the bolts which tighten the cable bands. While this decrease in itself is slight and not sufficient to loosen the clamps, it will be advisable to examine the tension in the bolts after the lower deck is in place and to tighten them where found necessary.

LOADS AND FORCES

Dead Load

As originally built the dead load per lin. ft. of center span was 21,300 lbs. In subsequent years a lower lateral bracing system was added to the deck structure as well as rails and supports for the maintenance platforms. These changes added 1,600 lbs., bringing the total dead load per ft. of bridge to 22,900 lbs.

The proposed lower deck will have an estimated dead load per lin. ft. of 2,600 lbs. Thus, if no modifications in the existing sidewalk were contemplated the total dead load per lin. ft. would be 25,500 lbs. To this a reserve of about 800 lbs. should be added for possible future resurfacing of the concrete pavement, ducts, water pipes, etc.

The stresses in the main carrying members resulting from such a dead load of 26,300 lbs., combined with the other forces, would be in excess of what we deem to be within acceptable conservative limits. However, it is possible to reduce this figure by certain changes in the existing structure which have no adverse effect upon its usefulness.

As previously mentioned the existing ten foot wide sidewalks are to be reconstructed to effect an estimated reduction in weight of about 1,000 lbs. per lin. ft. of bridge.

The construction of the lower deck will obviate the need for the interior traveling maintenance platform. Removal of the rails for this platform and their supports will further reduce the dead load by 150 lbs. per lin. ft. of bridge.

With these changes the total design dead load of the double-deck suspended structure will be about 25,100 lbs. per lin. ft. of bridge.

Design Live Load for Cables, Anchorages and Towers

The maximum axial stresses in the cables, anchorages and towers are obtained when the two side spans as well as the center span are loaded with the design live load. For maximum combined axial and bending stresses in a tower the design live load is extended over the center span and the far side span only. In the original design of the bridge the following uniform live load per lin. ft. of bridge was assumed.

500 lbs. on each lane of the six lane roadway	3,000 lbs.
500 lbs. on each of the two ten ft. sidewalks	1,000 lbs.
Total original design live load per lin. ft. of bridge	4,000 lbs.

The above live load is much too conservative when compared with the design loads adopted for other recently built long-span suspension bridges and in the light of more recent studies of vehicular traffic patterns and pedestrian use of such bridges. We consider the design live load per lin. ft. of bridge given below to be conservative for the combination of mixed five lane traffic on the upper deck combined with automobile traffic on the four lanes of the proposed lower deck.

400 lbs. per lane of upper deck	2,000 lbs.
175 lbs. per lane of lower deck	700 lbs.
100 lbs. on each of the two 10 ft. sidewalks	200 lbs.
Total uniform live load per lin. ft. of bridge	2,900 lbs.

The above figure includes impact and reductions for multiple lane loads in accordance with design practice for long span bridges.

Design Live Load for Stiffening Trusses

The maximum stresses in the stiffening trusses are produced by a live load extending over a restricted length. In accordance with established practice, as applied to long span suspension bridges, a higher intensity of live load

is properly used for such loadings. For the combined traffic over the two decks an intensity of 3,250 lbs. per lin. ft. of bridge is considered to be conservative.

Design Load for Deck Structure and Suspenders

For the review of stresses in the existing members of the deck structure and the design of the lower deck the Standard Specifications for Highway Bridges of the American Association of State Highway Officials (1961 Edition) have been followed. Five lanes of H20-S16 loading on the upper deck and four lanes of H10 loading on the lower deck have been assumed. Multiple lane reduction and impact factors are in accordance with the above mentioned Specifications.

Temperature Changes and Static Wind Forces

The addition of the proposed lower deck will have no effect upon the stresses and deformations of the bridge structure produced by temperature changes. The structural elements of the lower deck are designed to keep the additional area exposed to wind to a minimum. Because of the location of the lower deck well within the existing deck structure, this effect is very small.

Aerodynamic Action

In connection with the design studies made for the Verrazano-Narrows Bridge a program of comparative wind tunnel tests with models of sections of the suspended deck structure of various long-span suspension bridges was carried out. It included section models of the Golden Gate Bridge before and after the addition of the lower lateral system. The latter model was modified so as to simulate the effect of the addition of the lower deck.

In the wind tunnel the models representing the various conditions of the Golden Gate Bridge were subjected to wind velocities corresponding to 20 to 100 miles per hour on the prototype with angles of attack varying between 15° above and 15° below the horizontal.

The tests of the model with the lower deck and those of the model representing the present structure with bottom lateral system indicated no significant difference in behavior. Both confirmed the markedly improved effect of the lower lateral system.

Earthquake Effect

The influence of the lower deck on earthquake effects is insignificant. As at present the bridge will be adequately resistant against such action after the lower deck is in place.

STRESSES IN SUSPENSION SPANS

Stresses in Wire Cables

In the original design of the Golden Gate Bridge a very conservative permissible axial stress of 82,000 lbs. per sq. in. was adopted. In the design of the Verrazano-Narrows Bridge in New York it was considered proper to adopt 87,000 lbs. per sq. in. A large number of tests made previously proved the cable wire to be of uniformly excellent quality exceeding the specified minimum strength by 6% to about 235,000 lbs. in the average. The design live load on this bridge represents only 11.5% of the total load per lin. ft. Under these circumstances it was considered that, should it ever become necessary, the permissible axial cable stress could conservatively be raised to 90,000 lbs. per sq. in. This bridge provides for twelve lanes of unrestricted mixed vehicular traffic. While the probability of all spans loaded with an unduly large proportion of heavy trucks is extremely small, it had, nevertheless, to be taken into account in establishing the maximum permissible unit stresses. On the Golden Gate Bridge with the lower deck restricted to light vehicles the proportion of heavy trucks will probably be considerably less, so that a somewhat higher permissible stress may still be considered reasonably conservative. The maximum axial stress as computed for the double deck structure will be about 91,500 lbs. per sq. in.

Stresses in Anchorage Chains

The anchorage chains in the Golden Gate Bridge are composed of forged and heat treated eye-bars. The critical strength properties of these links are the yield point and ultimate strength, both determined from tests on full size bars.

The minimum values attained during tests of individual bars were 50,000 lbs. per sq. in. for the yield point and 79,700 lbs. per sq. in. for the ultimate strength.

The originally specified permissible axial design stress was 27,000 lbs. per sq. in. For the fully loaded double deck structure the maximum axial stress in the eye-bars will be about 28,700 lbs. per sq. in., inclusive of temperature effects or only 6% over the original design stress. The standard Specifications of the American Association of State Highway Officials permit a dead plus live load axial stress of 27,000 lbs. per sq. in. for low alloy material with a yield point of 50,000 lbs. per sq. in. and higher permissible stresses when temperature effects are included. The computed stress of 28,700 lbs. per sq. in. is therefore considered to be well within reasonable safety limits.

Stresses in Stiffening Truss Chords

The most critical stresses in the chords of the stiffening trusses result from a combination of the dead load, a live load of restricted length, temperature and lateral wind force. Maximum chord stresses are produced by combinations of either dead load, live load, temperature and one half wind or by dead load, one half live load, temperature and wind. For either of these combinations permissible unit stresses of 1.33 times the normal stresses are specified. For silicon steel these increased permissible stresses are 25,900 lbs. per sq. in. for compression on gross section and 32,000 for tension on net section.

The calculated maximum stresses in the chords of the double deck Golden Gate Bridge are within the above permissible unit stresses.

Stresses in Tower Shafts

The tower shafts of the Golden Gate Bridge are columns of exceptionally large sections composed of a multitude of individual rolled plates and shapes. The material is partly carbon steel, for which a yield point of 36,000 lbs. per sq. in. was specified, and partly silicon steel with a specified yield point of 45,000 lbs. per sq. in. The average yield point values obtained from specimen tests for all shapes were 40,800 lbs. per sq. in. for carbon steel and 52,400 lbs. per sq. in. for silicon steel.

For a combination of dead load, live load of unrestricted length and temperature change the originally specified maximum permissible axial stress for silicon steel was 18,000 lbs. per sq. in.

For the fully loaded double deck structure and specified temperature the calculated maximum axial stress at a silicon steel section eighty feet below top of tower is 19,200 lbs. per sq. in. This is 6% above the specified permissible stress but in view of the nature of the loading and the considerably higher specimen test yield point values mentioned above this stress is well within reasonably conservative limits.

Stresses in Suspenders

The calculated stresses in the suspenders for the two deck structure are well within the originally specified permissible stresses.

Stress Condition in Other Parts of Suspension Spans

The tower piers, pylons, anchorage blocks and their foundations have sufficient strength and stability to safely sustain the additional forces which would be imposed by the addition of a lower deck.

DEFLECTIONS AND THEIR EFFECT ON NAVIGATION CLEARANCES

The theoretical design deflection at the center of the main span caused by a combination of live load and a temperature rise of 110° F., used for calculating the stresses, is about 15 feet. This is about five feet below the original design clearance of 220 feet above mean high water level at the center.

For a realistic appraisal of the deflections of an existing structure with two decks a maximum design highway live load restricted to the center span combined with a theoretically extreme temperature will actually never develop.

A realistic live load combined with a maximum temperature of as high as 90° F., will not cause encroachment on the clearance of 220 feet at the center.

FEASIBILITY OF ADDITION OF LOWER DECK ON APPROACH VIADUCTS

The plans of the existing structures of the San Francisco and Marin approach viaducts were reviewed to ascertain the feasibility of adding a lower deck for four lanes of automobile traffic below the existing sixty foot wide roadway.

It is our conclusion that such a lower deck can be erected on the approach spans. All reconstruction work can be done below the existing roadway and without interference with normal flow of traffic.

COMPARISON WITH LOWER DECK FOR RAPID TRANSIT

In its report of April 1962 the Engineering Board of Review appointed by the Golden Gate Bridge and Highway District to investigate a proposed installation of rapid transit trains on the Golden Gate Bridge concluded that such installation was not advisable on the ground that it would encroach on the safety and length of life of the structure.

It appears therefore appropriate to point out the major differences between the addition of a lower deck for two rapid transit tracks and one for a four-lane roadway for automobiles.

In the case of rapid transit on the lower deck the axial stress in the wire cables was calculated by the Board of Review to be 97,500 lbs. per sq. in., nearly 12,000 lbs. or 12% higher than what the Board considered to be a conservative limit for this traffic.

For automobiles the maximum calculated stress is 91,500 lbs. or only one and one-half percent higher than the conservative limit laid down by the Board

for vehicular traffic. This is 40% of the average tested strength of the cable wire and still provides an ample margin of safety. Similar differences apply to the anchorage chains.

The most critical stresses from rapid transit in the tower shafts exceed the permissible stresses by up to 14%, whereas those from automobiles are only 7% above the conservative permissible stresses.

Impact effects from rapid transit trains on the major parts of deck struts, the stiffening truss and suspenders are appreciable, whereas those from automobiles are practically negligible. Fatique studies and tests made in connection with the investigation by the Engineering Board of Review indicated that rapid transit may appreciably shorten the life of the structure because of frequent and large stress fluctuations from the passage of trains, whereas in the case of auto traffic no such effect was indicated.

Rapid transit required strengthening of the stiffening trusses to various extents. Under traffic on the upper deck and temperature variations this would be an extremely difficult and time consuming operation with uncertain results. In the case of automobiles no alterations in the stiffening trusses are necessary.

The existing deck structure would, in the case of rapid transit, require extensive and difficult changes, possibly complete dismantling and replacement of the lower lateral system. Compared to these changes those proposed for automobile traffic are minor and very simple. The lower lateral system would not be affected.

Rapid transit would necessitate changing the expansion dams in the upper deck which could not be done without serious interference with the traffic. For automobile traffic this change is not necessary.